MEASURING DYNAMIC IMPACTON AGED TIMBER BRIDGES; SOME EXPERIMENTAL OPTIONS

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ABSTRACT

There are thousands of aged timber beam bridges on local roads in New South Wales (NSW) and because of deterioration their safety levels are unknown. To identify a bridge safety level requires structural performance measurement, preferably with a Structural Health Monitoring (SHM) system, so that any significant temporal change can be quickly identified. There is a need, however, to identify sensors and systems that can be used to monitor the dynamic impact of loads moving at highway speeds that are of adequate performance and of a cost that is a small fraction of the structures' value. Three measurement systems are considered: a high speed camera system to enable the establishment of base-line performance; a laser sensor system to enable accurate validation of other measurement systems on in-service structures; and a system comprising accelerometers to provide a relative motion record of components compared to the motion of a main girder.

KEYWORDS

Aged timber beam bridge, dynamic impact, mid-span deflection, laser, high speed camera, accelerometer.

INTRODUCTION

In the 19th and 20th centuries thousands of timber beam bridges were built on local roads in New South Wales (NSW) Australia; hundreds of thousands worldwide. Over two thousand are extant on local roads in NSW (Roorda, 2006); one example is Horton's Creek bridge on the Armidale Grafton road in NSW as shown in Figure 1, and another is Munsies Bridge as shown in Figure 2. Both of these structures carry some heavy loads and numerous light loads daily. Many have an unknown level of deterioration and their safety levels are unknown (Howard, 2009). Economic maintenance of these bridges is, therefore, an ongoing concern for the owners of regional bridges.

Figure 1: Hortons Creek Bridge, Armidale Road, NSW

Figure 2: Munsies Bridge, Gostwyck, NSW

In most of these bridges the components are bolted and spiked together with the elasticity of the joints being rarely well defined and often temporally changeable. Girders will sometimes twist and sometimes not depending on the position of the vehicle loading. Loose bolts will sometimes bind and freeze and other times move freely. To accurately determine the level of nodal coupling of such joints under load requires a method that includes the continuous measurement of every joint. A further measurement complication is that the level of nodal coupling will vary dependent on the traverse speed of any vehicle loading those joints. There is thus a deflection that is caused by a static load and an increased deflection that is caused by a moving dynamic load. Such an increased deflection is termed the ‘impact factor’ and Bakht & Pinjarkar(1989) identified that “impact factor is not a tangible entity susceptible of deterministic evaluation”; a reference more recently cited by Mufti (2001). While it may be straightforward to create a repeatable deterministic solution or even a Finite Element Model (FEM) solution for a new tightly constrained Timber Bridge it is more difficult to do so for a structure that can vary unpredictably in performance week by week. The effect of impact by an individual vehicle on a specific bridge can, therefore, only be determined by direct measurement of aged timber beam bridge girders with poorly constrained decks and to ensure the probability of failure is updated daily, continuous measurement with a Structural Health Monitoring (SHM) system is required.

In order to implement an SHM system on an aged timber bridge of unknown performance a baseline performance must be established and to achieve this the bridge must be measured under differing loading conditions. This baseline performance can then be compared with subsequent measurements to determine if temporal changes have occurred. Such temporal changes can then be used, particularly in the case of aged timber structures, to provide a Structural Safety Evaluation (SSE) as discussed by (Chan *et al*., 2011). To sense such temporal changes there are several possible systems that can be used but the ones considered here are:the measurement of the absolute mid-span deflection using both a high speed camera(Moore, 2013) and laboratory quality laser distance sensors; and the use of accelerometers to determine the relative movement of bridge girders and components.

THE MEASUREMENT OF MID-SPAN DEFLECTION

Deflection Recording With a High Speed Camera, Staff and Vernier

Deflections caused by traversing vehicles of known mass over a test bridge (Munsies Bridge, Gostwyck NSW)were recorded using a high speed high quality camera (Casio EX-FH25)(Moore, 2013). The camera was capable of recording at 120 frames per second (fps), 240 fps and 480 fps, with an upper speed of 1000 fps; 120 fps were used in this test. The camera and software were calibrated by comparing the recorded images with the moving sweep-hand of a standard clock. The software used to interpret the images was AVS Video Editor 5.2 sourced from Online Media Technologies Ltd, UK. The recordings from the video camera were used to identify the instantaneous mid-span deflectionsfrom a test vehicle moving across the bridge as measured with a graduated scale and vernier (refer Figures 3 and 4) .

Figure 3: Graduated scale, staff and vernier

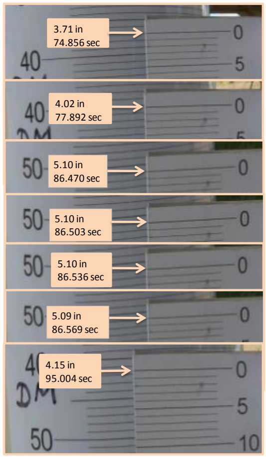
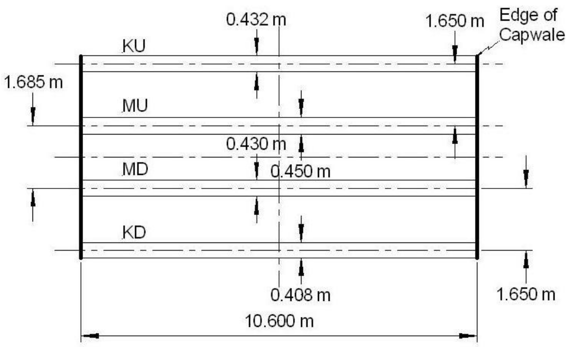


Figure 4: Close up view of some graduated scale and vernier measurement examples

The plan of Munsies Bridge, Gostwyck, NSW,is shown in Figure 5 and the cross-section in Figure 6. The span was 10.6 m and the four girders of various diameters in the range 400-450 mm. The bridge was originally constructed in 1938 and is still in operation. It is currently recommended by the Roads and Maritime Services (RMS), NSW, that a replacement bridge girder should be of F27 hardwood, Group S2, Durability class 2 to AS1720 with a minimum diameter of 450 mm (RTA, 2008: Section 1.9.1.2), but manyin-service girders do not comply with this recommendation. To determine the in-service girder Modulus of Elasticity (MoE), the mid-span deflection was measured under a known load. This was achieved by placing graduated scales under the mid-spans and attaching staffs with vernier scales to the mid-spans. A test vehicle of known axle load (refer Table 1 and Figure 7) was then driven across the span and the mid-span deflections recorded. Two models, one a finite element model (CSI, 2010) and the other a spread sheet model, were used to determine the effective values of girder MoE.



Girder nomenclature: K ~ kerb;
U ~ upstream; M ~ main; D ~ downstream

Figure 5: Plan of Span 4 girder layout

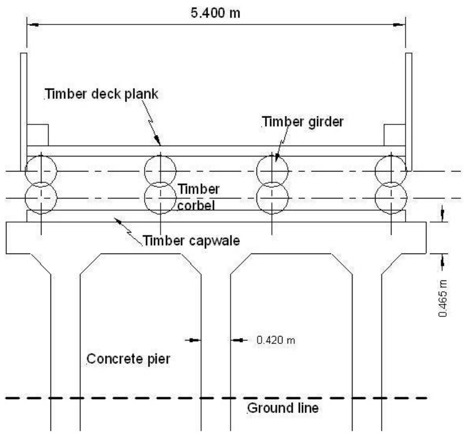


Figure 6: Cross section of Span 4

Figure 7: Test vehicle on bridge

Table 1: Test vehicle characteristics

Parameter	Value
Front axle load	41 kN
Rear axle load	124 kN
Axle spacing	4.3 m

To validate that the deflection data was a reasonable reflection of the applied load, a continuous record was made of the test vehicle traversing Span 4. The combined influence line was computed for both axles of the test vehicle (refer Table 2) and is shown in Figure 7 (smooth light grey line) The impact factor (Mufti, 2001: Equation 2.4) as determined in Table 5 and Figure 8 was 0.14.

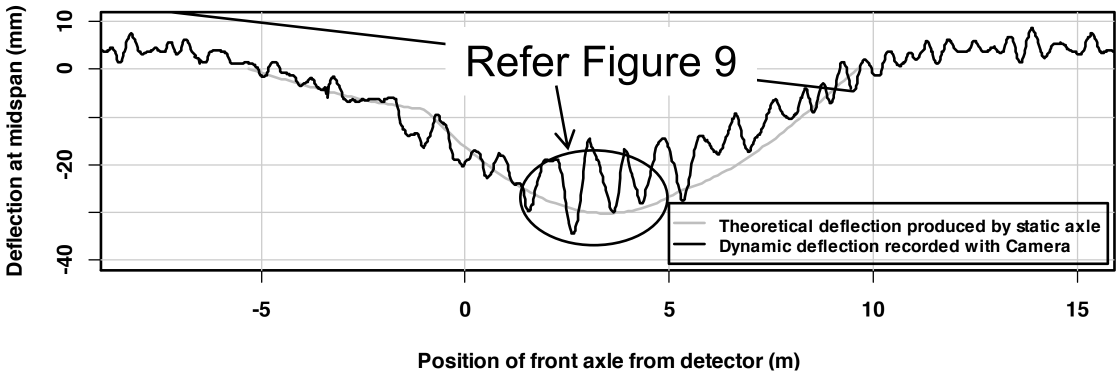


Figure 8: Dynamic influence line at mid-span produced by test truck on Span 4

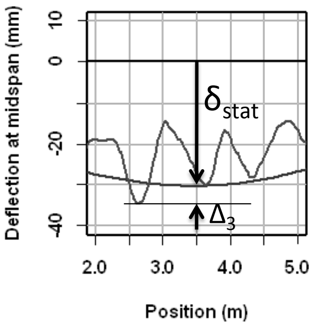


Figure 9: Loaded span, static and dynamic deflection

Table 2: Impact factor of loaded span

Parameter	Description	Magnitude
δ_{stat}	Peak static deflection of loaded span	30.1
Δ_3	Peak dynamic deflection of loaded span	4.2
I	Impact factor	0.14

Deflection Recording With Laser Distance Sensors

To validate other in-service measurements a high quality measurement system is required and to test such a system a Tokyo Sokki Kenkyujo TMR 200 Multi-recorder was tested in a laboratory situation. The first requirement is that samples can be recorded at sufficient speed to determine the peak mid-span deflection of a timber bridge girder. If a point load is one half a metre either side of the mid-span then the mid-span deflection will be within 5% of the peak value (Moore, 2009, Appendix C). At a traverse speed of 100 $km.h^{-1}$ the time for such a point load to move half a metre is 18 millisecond. A logging system capable of recording at 200 samples per second or higher (5 milliseconds between samples) is thus an appropriate system to use to record dynamic deflection in sufficient detail to accurately identify the peak deflection. The test system that was setup in the laboratory comprised: a simply supported 5 metre aluminium beam (plank); a TMR 200 Multi-recorder together with a TMR-231 Voltage/Thermocouple unit and a computer interface sampling at five millisecond intervals; and a Micro-Epsilon optoNCDT 1402 laser sensor to measure the deflection near mid-span when deflected by an impulse load (refer Figure 10). The beam was loaded by stepping onto it and causing it to vibrate at its natural frequency. The resultant transient was as shown in Figure 11 and a cycle of the transient shown in Figure 12. The measured period was 1.9 Hz with about 100 samples per cycle.

Figure 10: Beam used to test vibration logging system

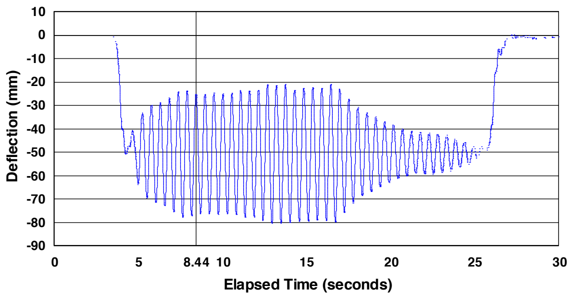


Figure 11: Transient deflection of beam

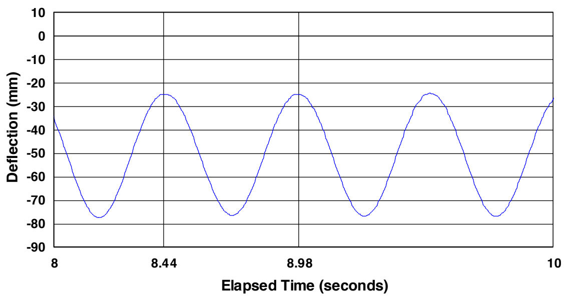


Figure 12: One cycle, between 8.44 and 8.98 seconds

RECORDING RELATIVE MOTION WITH ACCELEROMETERS

To enable the choice of which accelerometers can be used in-service on timber beam bridges it is required that the sensor sensitivity must be first be established. This was achieved by examining the case of a simply supported beam as shown in Figure 13, of length l, with a concentrated load, P, applied at a distance x from the support. The deflection of the beam, δ, at a distance d from the support, providing $d > a$is then given (Benham & Crawford, 1987, p. 181: Equation 7.31)as shown by Equation 1. If the deflection is doubly differentiated then the vertical mid-span acceleration, a_{vp}, is obtained as shown by Equation 5.

$$\delta = \frac{P}{6\,E\,I} \times \frac{d(L-x)}{L} \times (2xL - x^2 - d^2) \tag{1}$$

where:

P	Load (N)
d	Distance from support to measured deflection
L	Beam span (m)
x	Distance from support to Load
E	MoE (GPa)
I	Second moment of area (m^4)

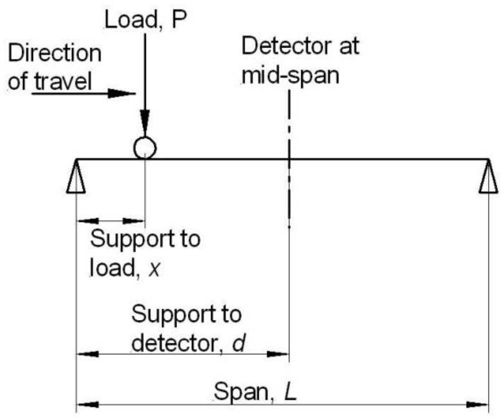


Figure 13:

Then if $d = \frac{L}{2}$:

$$\delta = \frac{P}{48\,E\,I}(4x^3 - 12x^2L + 9bL^2 - L^3) \tag{2}$$

If the load traverses the beam at a speed v and takes the time t to traverse the distance x then:

$x = vt$, and

$$\delta = \frac{P}{48\,E\,I} \times (4v^3t^3 - 12Lv^2t^2 + 9L^2vt - L^3) \tag{3}$$

The vertical velocity of the beam at mid-span is given by:

$$\frac{\partial\delta}{\partial t} = \frac{P}{48\,EI} \times (12v^3t^2 - 24Lv^2t + 9L^2v) \tag{4}$$

Now the peak vertical acceleration, a_{vp}, will occur when the vehicle is at the mid-span, hence:

$$a_{vp} = \frac{\partial^2\delta}{\partial t^2} = \frac{Pv^2L}{4\,EI} \tag{5}$$

The peak mid-span vertical acceleration of the beam is thus proportional to the load and the square of the traverse speed of the load across the beam. A typical timber beam bridge with four girders will support 32% of the load on one main central girder (Moore *et al*., 2012, Table 1)and a typical set of acceleration values is given in Table 3. The first two cases (I & II) evaluated are for a light load, 3 tonne, traversing at medium and high speed (40 $km.hr^{-1}$ and 100 $km.hr^{-1}$). The typical maximum axle load in NSW is about 20 tonne and this is evaluated at a speed of 60 $km.hr^{-1}$(Case III) and to compare the data reported by Feltrin, Steiger, Gsell, Gulzow, & Wilson(2010) Case IV is evaluated for a point load of 40 tonne.

A sensor is, therefore, required that can be used to detect accelerations in the range 0.005G to 0.5G. A practical example of measurements of this type has been reported by Feltrin et al.(2010). They reported that a newly constructed wooden trough bridge loaded with a 40 tonne vehicle created beam mid-span vertical accelerations in the range 0.1 ms^{-2} rising to 1 ms^{-2} for vehicle traverse speeds of about 10 kmh^{-1} to 60 kmh^{-1}. While the structure they report is different to a four girder timber beam bridge the measured accelerations are indicative of a timber structure designed to carry similar loads on local roads and, therefore, expected to undergo comparable accelerations to those that might be expected on a timber beam bridge structure.

Table 3: Calculation of vertical acceleration of beam at mid-span for a 3 tonne point load at 40 $km.hr^{-1}$ and 100 $km.hr^{-1}$ (Case I & 2), and also fora 20 tonne and a 40 tonne point load at 60 $km.hr^{-1}$(Case III&IV)

Case	I	II	III	IV	Parameter (units)
P	3	3	20	40	Total load (tonne)
P	29	29	196	392	Total load (kN)
0.35P	9418	9 418	62784	125568	Load per central girder (N)
l	10	10	10	10	Beam span (m)
v	40	100	60	60	Vehicle traverse speed ($km.hr^{-1}$)
v	11.1	27.8	16.7	16.7	Vehicle traverse speed ($m.s^{-1}$)
E	20	20	20	20	MoE (GPa)
I	0.002	0.002	0.002	0.002	Second moment of area (m^4)
a_{vp}	0.07	0.45	1.1	2.2	Vertical beam acceleration ($m.s^{-2}$)
a_{vp}	0.007 G	0.046 G	0.1 G	0.2 G	Vertical acceleration as a fraction of G (9,8 $m.s^{-1}$)

DISCUSSION

While it has been reported (Moore, 2013; Moore *et al*., 2013) that continuous monitoring of timber beam bridges is viable the possibility of lower cost systems has not been reported. While a monitoring system to record the mid-span deflection of a single girder might cost less than five thousand dollars this cost rises significantly if all the girders on a multi-span bridge need to be monitored. A single girder monitoring system such as the camera based system discussed above can be economically used to determine base line performance and a camera and laser based system (Moore *et al*., 2013) can be used to continuously monitor several girders. These systems can be validated when used in-service by using laboratory laser distance sensors as also indicated above. However, the cost of these laboratory sensors is prohibitive, for local councils, when twenty to thirty sensors are required to monitor one bridge.

An alternative approach is to monitor the mid-span deflection of a few girders with a device such as the Bridge Deflection Meter (BDM) described by Moore(2013) and to monitor the remainder with accelerometers. While laboratory style accelerometers can be high cost at several thousand dollars lower cost proprietary devices can be constructed for about one hundred dollars (Citation: personal experience). Accelerometer integrated circuits are available for less than one dollar, they just need to be packaged appropriately with suitable data interfaces for in-service use on timber bridges. Twenty devices at \$100 is thus more economical than 20 at \$5000. To attempt to utilise such accelerometers to monitor girder deflection is not straightforward as discussed by (Arraigada & Partl, 2006). Identifying the required constants of integration can require excessive repetitive calibration experiments. However, the relative component motions can be compared by comparing their accelerations. The motions can firstly be compared amongst all the girders and then with the base line measurements. Since these types of comparisons can be done in real time a structural health monitoring system can be constructed that could be used to identify significant temporal change at costs that are a small fraction of the structural value.

CONCLUSIONS

Because of the ongoing need to economically determine the health of aged timber beam bridges, viable sensing systems need to be identified that can be effectively used on such structures. Three different types of sensing systems have been presented that are suitable for use to evaluate timber bridge structural health. The use of these types of systems should enable Local Government Engineers to effectively establish baseline performance and the occurrence of any ongoing temporal change. It is also anticipated that these sensors will be used in further research to establish the validity of determining the baseline and ongoing temporal structural health of aged timber beam bridges. It was not possible, in this paper, to compare the results, using the three methods from one in-service bridge but such a comparison will be made in a future paper as soon as data is available. To achieve this we are interacting with Local Government in our area to establish suitable measurement opportunities.

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